EVALUATION OF DESIGN PARAMETERS FOR SHALLOW FOUNDATIONS ON ALLUVIAL SOILS

A Thesis Submitted
in Partial Fulfilment of the Requirements
for the Degree of
MASTER OF TECHNOLOGY

By
UMESH CHANDRA JAIN

to the
DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR

AUGUST, 1978

DEDICATED

TO

MY BELOVED PARENTS

CE-1978-M-DAI-EVA

N... ...IBRARY
... No. A 54908.
... 78

CERTIFICATE

Certified that the thesis entitled "Evaluation of Design Parameters for Shallow Foundations on Alluvial Soils" is a record of work carried out by Mr. Umesh Chandra Jain under my supervision and that it has not been submitted eslewhere for a degree.

Yudhbir
(DR. YUDHBIR)
Professor
Department of Civil Engineering
Indian Institute of Technology, Kanpur

August, 1978

14.8.78

ACKNOWLEDGEMENTS

"Authorship of any sort is a fantastic
indulgence of the ego. It is
well, no doubt, to reflect
on how much one owes
to others."

J.K. Galbraith, The Affluent Society
Houghton-Mifflin Co., Boston, Mass.

I have immense pleasure in expressing my deep sense of gratitude and sincere thanks to Dr. Yudhbir, my thesis supervisor, who has initiated me into the problem, and has inspired me with his great interest and guidance time and again.

I thank Mr. Z.C. Bendale for carefully reading the manuscript and for offering valuable suggestions. I thank Mr. P. Singh for letting me use his electronic calculator.

Thanks are due to Mr. K.V. Lakshmidhar and others in the Soil Mechanics Laboratory for their help in the laboratory.

The neat and efficient typing by Mr. V.K. Saxena and Mr. G.S. Trivedi is very much appreciated.

U. C. Jain

CONTENTS

ABSTRACT

The in-situ soil parameters for the design of shallow foundations on alluvial deposits have been evaluated on the basis of the field tests conducted in the south-western part of the Indian Institute of Technology Kanpur Campus.

The fluctuation in the ground water level in this area is reported for a period of 3 years (1975-78). The soil profile within the zone of influence of shallow foundations as constructed in this area has been well defined.

The effect of ground water level and the moisture content (the degree of saturation) on the soil parameters has been studied. The variation in allowable bearing capacity with the plate size is presented.

Design parameters for the shallow foundations are recommended. It has been recommended that the allowable bearing capacity in these soils can be safely increased to 14 t/m^2 as against 8-10 t/m^2 being currently used in foundation design.

LIST OF SYMBOLS

D_w	:	Depth of ground water level from ground surface
D	:	Depth of foundation level from ground surface
B	:	Diameter of test plate
	:	Width of foundation
L	:	Length of foundation
GL	:	Ground level
FL	:	Foundation Level
N	:	Standard penetration number (Blow count per 30 cm of penetration of standard split spoon)
w_n	:	Natural moisture content
w_l	:	Liquid limit
w_p	:	Plastic limit
I_p	:	Plasticity index
γ_t	:	Total (Bulk) density of soil
γ_d	:	Dry density of soil
Q	:	Load in tons (t)
A	:	Area of test plate
P	:	Stress (=Q/A) in t/m^2

p	:	Normalized load (=Q/B) in t/m
Δ	:	Settlement in mm.
q_{all}	:	Allowable bearing capacity
q_{ult}	:	Ultimate bearing capacity
C_w	:	Correction factor for ground water level
q_c	:	Cone resistance
S_u	:	Undrained cohesion
K_s	:	Modulus of subgrade reaction
E_u	:	Undrained elastic modulus
E'	:	Drained Elastic modulus
ν	:	Poisson's ratio
F	:	Factor of safety
S_i	:	Initial settlement
S_t	:	Total settlement
S_{1-D}	:	Settlement by one dimensional analysis
m_v	:	Coefficient of volume compressibility
ν'	:	Drained Poisson's ratio

LIST OF FIGURES

Figure No.

CHAPTER 1

INTRODUCTION

Nearly every Civil Engineering structure - building, bridge, highway, tunnel, wall, tower, canal or dam is to be founded in or on the surface of the earth. For satisfactory performance, each and every structure must have a proper foundation. A detailed soil investigation at site is essential before any foundation can be designed. The main object of the soil investigation at site is to provide the designer with as much information as possible about the existing conditions at the site such as the exposed overburden, position and fluctuations in ground water level, vegetation and other geological features of the area, and the subsoil conditions below the proposed structure.

The present study is a link in a chain of continuous effort that is being put in at I.I.T. Kanpur in the field of soil investigations to determine the relevant soil parameters for the design of shallow foundations on alluvial deposits in and around Kanpur City.

The term 'shallow foundation' refers to a structure that is supported by the soil lying immediately beneath the structure. Individual footings, usually rectangular in plan, are the most common shallow foundations for colums, whereas strip footings are used to support walls. In some cases where the sum total area of the individual footings becomes more than half of the total covered area, mats are preferred over individual footings.

The Kanpur City is located on the banks of the river Ganga. The soil around this area is a part of alluvial deposits laid down by Ganga. These stratified deposits have different layers of sand, silt and clays at different depths at different locations, and are highly variable in nature.

The theoretical advances in the area of soil mechanics are far ahead of our knowledge about in-situ properties of natural soils. The best alternative seems to be the field testing to obtain the relevant soil parameters needed for designing the foundations. Though the field testing has its own difficulties of transporting the equipments etcetera, the laboratory tests conducted on the so called undisturbed samples give soil parameters

far from being representative of field-soil behaviour.

With this in view, the plate load tests have been conducted at the site of married students' hostel in I.I.T. Kanpur Campus. The plate load test has a serious limitation to its applicability. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger in size, the settlement and resistance against shear failure depends on the properties of a much thicker stratum. Thus the results of the test are likely to be misleading, if the character of the soil changes at shallow depths. So, the standard penetration tests have also been carried out at the same site and the remoulded soil samples have been collected for soil identification purposes. However, based on the present and past investigations the soil profile within the zone of influence of shallow foundations, (located at 1m depth with width = 1m) as constructed in this area, has been well defined. This will help to interpret the plate load test data with a reasonable degree of confidence.

Based on plate load tests and the standard penetration values the design parameters, i.e. allowable bearing capacity, coefficient of volume compressibility, drained and undrained Young's modulii etc. have been obtained and compared with available data for similar soils in the literature. A three year data on ground water fluctuations in this area is reported.

CHAPTER 2

TEST DETAILS

2.1 General :

A brief account of the test site is given. The details of the tests conducted are briefly described in this chapter.

2.2 Location of Test Site :

Since the present study is a part of continuous work carried out earlier by C.K. Jain (1977) and N.K. Srivastava (1977), the test site has been chosen to be very near to their test sites so as to facilitate the proper comparisons of the test results.

The test site is located at Married students' hostel near Type V houses. Fig. 2.1 shows the location of the test site in IIT Kanpur Campus. This figure also indicates the relative positions of Jain (1977) and Srivastava (1977) test sites in that area. The position of two ground water level observation wells of 3m and 4m depth is also shown in this figure.

2.3 Ground Water Level :

The area around the test site is one of the many low lying areas in I.I.T. Kanpur campus which easily get water logged during rainy seasons. To add to the misery, there are other active water sources nearby, for example, a pond in front of Girls' Hostel and a Canal (Kalyanpur Distributory). All these result in bringing the ground water level very close to the ground surface.

The ground water level goes down significantly in dry season. A study on water table fluctuations carried over past three years shows that ground water level comes as near as 30 cm below ground surface during wet season and goes down as deep as more than 4m below ground surface in dry season as shown in Fig. 2.2. This rise and fall in ground water level causes considerable amount of variations in effective stresses on the soil particles. As the water table goes down, the soil gets subjected to suction pressure which results in development of negative pore pressure and an increase in the effective stresses. This makes the soil at shallow depths behave as a lightly over consolidated soil.

2.4 Details of Tests Conducted :

Eight test pits and three bore holes for conducting plate load tests and standard penetration tests respectively have been made at test site. Fig. 2.3 shows the relative positions of different test pits and bore holes at test site. These test pits and bore holes have been named as Pit 1 to Pit 8, and Bore hole 1 to Bore hole 3 respectively.

The following field tests have been conducted at site.

1. Plate load test
2. Standard penetration test
3. In-situ density test

Following are the tests conducted in laboratory on remoulded soil samples brought from the pits and bore holes.

1. Natural Moisture Content
2. Atterberg Limits
3. Grain Size Analysis

2.4.1 Plate Load Test :

The main purpose of this test is to evaluate the bearing capacity and the settlement of the soil. Field value of deformation modulus is also estimated from this test.

All the plate load tests have been conducted at the same depth of one meter from ground surface which is the usual foundation level for the shallow foundations. One meter deep test pits which are five times as wide as the test plate are made.

Reaction type of loading has been used for all the tests except for one test conducted with a 60 cm Ø plate in which case, a gravity type of loading is used as the loading frame is only 2m. wide. The loading frame consists of four hand driven helical augers which are driven into the ground at a distance of 2m centre to centre, and five channel sections which are fixed on the augers with nuts and bolts.

The test plate is placed at the bottom of the pit only after levelling the ground properly. The jack is mounted on the plate such that their centres coincide. Filler plates are provided on the jack to fill the gap between the jack and the channel of the loading frame. For tests conducted with test plates bigger than 30 cm Ø, a series of smaller plates are put concentrically between the test plate and the jack so as to ensure proper distribution of the load on the bottom plate. Three dial gauges are used to measure the settlement. The hydraulic jack is

lifted using a hand pump and loading is carried out in half ton intervals. Dial gauges are read for every load increment when the rate of settlement drops down to 0.02 mm/min.

2.4.2 Standard Penetration Test :

Standard penetration test conducted with the split spoon provides data about resistance of the soil to penetration in terms of **blow** count number (N) which is the number of blows per 30 cm of penetration of standard split spoon.

The bore holes are made using a 10 cm Ø hand-auger, operated by 3 to 4 men at a time, upto the required depth. The split spoon is driven for 45 cms into the ground by means of a 65 kg. weight with a free fall of 75 cms. The number of blows required for each of three successive penetrations of 15 cm each are recorded. The total number of blows required to penetrate the last 30 cms gives the value of N as the first 15 cm of drive is considered to be a seating drive.

The value of N has been found out at every half meter depth starting from 1m depth upto 4m below ground surface which are plotted in Fig. 2.3.

2.4.3 In-Situ Density Test :

In-situ density has been found out for all the 8 test pits using sand cone method. Knowing the weight of the sand occupying the hole from which the soil is scooped out and density of sand, in-situ bulk density of soil (γ_t) is calculated using the following expression.

$$\gamma_t = \frac{\text{Weight of soil scooped out}}{\text{Weight of sand in place of soil}} \times \text{Density of sand}$$

The test results are given in Table 3.1.

2.4.4 Natural Moisture Content :

Natural moisture content values have been found out for all the soil samples brought in plastic bags from the test site. These values are shown in Table 3.1 for test pits and in Fig. 2.6 for bore holes.

2.4.5 Atterberg Limits :

Both liquid and plastic limits of soils depend on the amount and type of clay in a soil and form the basis for the soil classification system for cohesive soils based on the plasticity tests.

The liquid limit and plastic limit tests have been carried out as per standard test procedures laid down

for the tests. The test results are given in Fig. 2.6. A plasticity chart has been drawn in Fig. 2.4, and the soils have been classified as CL and CI soils.

2.4.6 Grain Size Analysis :

Grain size analysis provides quantitatively the proportions by mass of the various sizes of particles present in the soil.

Grain size analysis using a 152 H type of hydrometer has been carried out as per standard test procedure laid down for the test, and particle size distribution curves have been plotted as shown in Fig. 2.5.

CHAPTER 3

TEST RESULTS AND THEIR INTERPRETATION

3.1 General :

The results as obtained from various tests are presented and interpreted in this chapter with a view to arrive at the design parameters for shallow foundations.

3.2 Plate Load Test :

Eight plate load tests have been conducted using different sizes of circular plates. Ground water level is noted down on the day of conducting the test. Table 3.1 gives the diameters (B) of the test plates used in various test pits along with the ground water levels (D_w), bulk density (γ_t), natural moisture contents (W_n) and dry densities (γ_d) for these tests.

TABLE 3.1

Pit No.	B cm	D_w m	γ_t gm/cc	W_n %	γ_d gm/cc
1	30	1.50	1.97	19.1	1.66
2	30	1.50	2.03	18.9	1.71
3	45	0.40	2.05	20.8	1.70
4	60	1.00	1.97	19.8	1.65
5	45	1.77	2.04	19.4	1.72
6	30	2.54	1.90	13.2	1.68
7	45	2.58	1.98	14.2	1.73
8	22.5	2.65	1.90	15.5	1.65

The hydraulic jack used for loading the plates has been calibrated using a 50 ton proving ring in the rock testing machine. The calibration chart is given in Fig. 3.1. The load as shown by the hydraulic jack pressure gauge is corrected, and this corrected load is used for further calculations.

3.2.1 Load-Settlement Characteristics :

Load-settlement data as obtained from the plate load tests have been plotted in Fig. 3.2. The number on the curves in this plot and in subsequent plots refers to the corresponding test pit number.

Fig. 3.3 shows plate load test data plotted in the form of stress, P (= Load/Area of the test plate) versus settlement.

An alternate way of presenting the plate load test data is given here. The total load (Q) on the plate is normalized with respect to the diameter (B) of the test plate, and this normalized load, p (=Q/B) versus settlement is plotted and presented in Fig. 3.4. This type of load normalization facilitates to represent the effect of ground water level and moisture content on the load deflection characteristics of the soil. The curves get lined

up in three distinct zones representing three different water table and moisture content conditions. As ground water level goes down, the curves shift upward which shows that the soil has become stiffer due to lowering in the water table and also the decrease in the degree of saturation.

The curve 8 does not appear to follow the general trend. Unfortunately, this test has been carried out on a filled up soil and not on a natural deposit. That it is so was indicated by a pipe line running along one edge of the test pit 8. Clearly, the area was dug up to lay the pipe line and had been refilled. This also appears to be so from a decrease in N-values observed in bore hole 1 from 1 to 2m depth as shown in Fig. 2.3.

Load-settlement data can be analysed in two different ways to estimate the bearing capacity of the foundation. ISI has outlined one method and the other method has been suggested by Kee (1972).

3.2.2 ISI Method :

In this method, a graph of stress (P) versus settlement is plotted on log-log scale as shown in Fig. 3.5. A distinct kink is observed in each plot which separates

the plastic yield settlement from elastic settlement. The stress at this point corresponds to yield stress approximately and is taken as the ultimate bearing capacity (q_{ult}). A factor of safety of 2 has been suggested by ISI to be applied on the ultimate bearing capacity so obtained to get the allowable bearing capacity (q_{all}). Table 3.2 gives the ultimate and allowable bearing capacity values as obtained by ISI method.

TABLE 3.2

ULTIMATE AND ALLOWABLE BEARING CAPACITY BY ISI METHOD

Pit. No.	Ultimate Bearing Capacity t/m^2	Settlement (Δ) at q_{ult} mm	Allowable Bearing Capacity (q_{all}) t/m^2
1,2	20	7.0	10.0
3	20	8.0	10.0
4	13	5.5	6.5
5	21	5.4	10.5
6	50	6.5	25.0
7	28	4.5	14.0

It is to be noted that ISI permits a settlement of 40 mm under allowable stresses whereas in this case,

the values of settlement obtained from the plate load tests are much smaller at the ultimate value of bearing capacity given by the ISI method. This suggests that the $q_{all.}$ thus obtained is likely to be conservative.

3.2.3 Kee's Method :

This method is applicable to the soils which have the load-settlement characteristics as hyperbolic in nature. A general equation of the following form can be written for such load-settlement characteristics.

$$y = \frac{x}{a + bx} \qquad (3.1)$$

Where, a and b are constants which can be determined by plotting x/y versus x and fitting a straight line to it. It has been found out that for a better fit to test data, this line should pass through the 70 % and 95% points. This straight line can be represented by the following equation.

$$\frac{x}{y} = a + bx$$

Where, a = the intercept on x/y axis

b = slope of the straight line

and, $1/a = \frac{dy}{dx}$ at $x = 0$

$1/b = y_{max}$.

Using this method, all the three plots shown in Figs. 3.2, 3.3 and 3.4 have been interpreted and are presented in the following sections.

3.2.3.1 Interpretation of Load Versus Settlement Plot :

For load (Q) versus settlement (Δ) curves of Fig. 3.2, in equation 3.1,

$y = Q$ and $x = \Delta$

so to find the constants, a and b, a plot of $\frac{\Delta}{Q}$ versus Δ has been plotted for all the curves of Fig. 3.2, and is shown in Fig. 3.6. The values of a, the intercepts and b, the slopes have been found out from Fig. 3.6 which are tabulated in Table 3.3.

TABLE 3.3

Pit No.	a mm/t	b 1/t	1/a t/mm	$\frac{1}{b} = Q_{max}$ t	$\frac{Q_{max}}{A} = q_{ult}$ t/m^2	$\frac{1/a}{A} = K_s$ t/m^2/mm
1,2	3.75	0.25	0.27	4.0	56.62	3.77
3	1.55	0.14	0.65	7.14	44.92	4.09
4	0.80	0.14	1.25	7.14	25.26	4.42
5	0.90	0.13	1.11	7.69	48.38	6.98
6	1.10	0.12	0.90	8.33	117.91	12.74
7	0.75	0.065	1.33	15.38	96.75	8.37

3.2.3.2 Interpretation of Stress Versus Settlement Plot :

For stress (p) versus settlement curves of Fig. 3.3, in equation 3.1,

$$y = p \quad \text{and} \quad x = \Delta,$$

so to find the constants, a and b, a plot $\frac{\Delta}{p}$ versus Δ has been plotted for all the curves of Fig. 3.3, and is shown in Fig. 3.7. The values of a and b have been tabulated in Table 3.4.

TABLE 3.4

Pit No.	a $\frac{mm}{t/m^2}$	b $\frac{1}{t/m^2}$	$1/b = P_{max} = q_{ult}$ t/m^2	$1/a = K_s$ $\frac{t/m^2}{mm}$
1,2	0.300	0.0165	60.61	3.33
3	0.215	0.023	43.48	4.65
4	0.240	0.0375	26.67	4.17
5	0.155	0.0205	48.78	6.45
6	0.085	0.0085	117.65	11.76
7	0.130	0.0085	117.65	7.69

3.2.3.3 Interpretation of Normalized Load Versus Settlement Plot :

For normalized load (p) versus settlement curves of Fig. 3.4, in Equation 3.1,

$y = p$ and $x = \Delta$,

so to find the constants, a and b, a plot of $\frac{\Delta}{p}$ versus Δ has been plotted for all the curves of Fig. 3.4, and is shown in Fig. 3.8. The values of a and b have been found out from Fig. 3.8 which are presented in Table 3.5.

TABLE 3.5

Pit No.	a $\frac{mm}{t/m}$	b $\frac{1}{t/m}$	1/a $\frac{t/m}{mm}$	$1/b = p_{max}$ t/m	$p_{max}\frac{B}{A} = q_{ult}$ t/m^2	$\frac{1}{a}\,\frac{B}{A} = K_s$ $\frac{t/m^2}{mm}$
1,2	1.275	0.0725	0.78	13.79	58.60	3.31
3	0.750	0.0600	1.33	16.67	47.18	3.77
4	0.575	0.0700	1.74	14.29	30.96	3.69
5	0.500	0.0575	2.00	17.39	49.26	5.66
6	0.350	0.0375	2.86	26.67	113.23	12.14
7	0.325	0.0325	3.08	30.77	87.13	8.72

3.2.3.4 Ultimate and Allowable Bearing Capacity :

The three plots, namely , $\frac{\Delta}{Q}$ vs Δ plot , $\frac{\Delta}{P}$ vs Δ plot and $\frac{\Delta}{p}$ vs Δ plot provide almost same values for ultimate bearing capacity as is clear from Table 3.3, 3.4 and 3.5. Any one of these plots may thus be used for interpretation of test data. The average values of ultimate bearing

capacity are presented in Table 3.6. Usually a factor of safety of 3 is applied on ultimate bearing capacity values so obtained to get the allowable bearing capacity. The allowable bearing capacity values are also tabulated in Table 3.6.

TABLE 3.6

ULTIMATE AND ALLOWABLE BEARING CAPACITY

Pit No.	Ultimate Bearing Capacity t/m^2	Allowable Bearing Capacity t/m^2
1.2	58.61	19.54
3	45.19	15.06
4	27.63	9.21
5	48.81	16.27
6	116.26	38.75
7	100.51	33.50

A correction factor (C_W) of the following form has been suggested by Peck to be applied on ultimate bearing capacity values to account for ground water level variations.

$$C_w = 0.5 + 0.5 \left(\frac{D_w}{D+B}\right) \qquad \text{(Eq. 3.2)}$$

for $D_w \leq D+B$

Where , D_w = Depth of ground water level below ground surface

D = Depth of foundation level

B = Width of the foundation

Using this expression, the water table correction factors (C_w) have been calculated for D = 1.0 m and D_w equal to the depth of water table on the day of testing, 1.0 m, 0.3 m and for water level at the ground level since the water table is known to rise upto and above the foundation level,and are tabulated in Table 3.7.

TABLE 3.7

Pit No.	B m.	Existing Water table on the test day m	C_w			
			For D_w=as in (3)	For D_w=1.0m	For D_w=0.30 m	For D_w=0
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1,2	0.30	1.50	1.0	0.88	0.62	0.5
5	0.45	1.77	1.0	0.84	0.60	0.5
6	0.30	2.54	1.0	0.88	0.62	0.5
7	0.45	2.58	1.0	0.84	0.60	0.5

It should be remembered that the equation 3.2 is normally used to correct allowable bearing capacity for expected rise of water level and is being checked here to see whether the plate load test data can be suitably corrected for rise of water level subsequent to testing (in case the test was not performed at water table as is recommended).

The corrected values of allowable bearing capacity for D_w = 0.3 m (the worst known case) are shown in Table 3.7(a). It should be noted that the high values of 24.03 and 20.10 t/m^2 for tests 6 and 7 respectively need to be further corrected for differences in natural moisture content with respect to saturation moisture content ($\simeq$ 20%). From Fig. 4.1 (d), the values of q_{all} for tests 6 and 7 after correcting for differences in natural moisture contents are 11.50 and 9.54 t/m^2 respectively.

TABLE 3.7 (a)

Pit No.	C_w for D_w=0.3 m	w_n %	q_{all} t/m^2
1,2	0.62	19.0	12.11
3	-	20.8	15.06
4	-	19.8	9.21
5	0.60	19.4	9.76
6	0.62	13.2	24.03/11.50*
7	0.60	14.2	20.1/9.54*

* Values after correcting for the natural moisture content with respect to saturation moisture content.

It would thus appear that in case tests are not done at the water table, the values thus obtained, if corrected for (i) differences in natural moisture content (with respect to saturation water content) and (ii) for rise in water level using equation 3.2 will yield results comparable with those appropriate for saturated and submerged condition. It may be mentioned that variation from 9.21 t/m^2 (table 3.7.a) for 60 cm Ø plate (recommended for design) after applying all these corrections are due to the size effect (as shown in Fig. 4.1 (b)) and the well known variation in the distribution of kankar in these soils.

3.2.3.5 Deformation Modulus :

The theory of elasticity provides the following expression for displacement (Δ) under a circular, rigid plate with a load intensity of P,

$$\Delta = \frac{PB}{E} (1-\nu^2) \times \frac{\pi}{4}$$

$$\therefore E = \frac{P}{\Delta} \cdot B\,(1-\nu^2)\frac{\pi}{4}$$

where, $\frac{P}{\Delta} = K_s$ = Modulus of subgrade reaction

and $\nu = 0.5$ for undrained case

so, $E_u = 0.59\ K_s\ B$ (Eq. 3.3)

The theory of elasticity also provides an expression relating drained elastic modulus with undrained elastic modulus, and it is :

$$E_u = \frac{3\ E'}{2(1+\nu')}$$

Srivastava (1977) has determined a value of 0.28 for ν' based on laboratory experiments on saturated block samples which is close to 0.25 suggested by Poulos (1975) for such stiff soils. For $\nu' = 0.28$,

$E' = 0.85\ E_u$ (Eq. 3.4)

The three plots, namely, $\frac{\Delta}{Q}$ vs Δ plot, $\frac{\Delta}{P}$ vs Δ plot and $\frac{\Delta}{p}$ vs Δ plot provide almost similar values for the modulus of subgrade reaction (K_s) as is shown in Tables 3.3, 3.4 and 3.5. The average values of modulus of subgrade reaction are tabulated in Table 3.8. The equation 3.3 has been used to calculate the initial elastic modulus values which are given in Table 3.8.

Normally secant modulus which is obtained at half the ultimate load, is used to specify the elastic modulus.

The secant modulus as obtained from the plate load test data and drained modulus as calculated using equation 3.4 are presented in Table 3.8.

TABLE 3.8

DEFORMATION MODULUS

Pit No.	K_s $\frac{t/m^2}{mm}$	Initial Elastic modulus t/m^2	Secant Elastic modulus t/m^2	Drained Elastic modulus t/m^2
1,2	3.47	614	296	252
3	4.17	1107	574	488
4	4.09	1448	708	602
5	6.36	1689	911	774
6	12.21	2161	1126	957
7	8.26	2193	1180	1003

3.3 STANDARD PENETRATION TEST :

This test gives the number of blows required to drive in the standard split spoon for a depth of 30 cm. The field N-values thus obtained are corrected for the overburden. Gibbs and Holtz (1957) have given the following

expression for overburden correction to be applied on field N-values to get corrected N-values.

$$\text{Corrected-N} = \text{Field-N} \times \frac{3.5}{p_o + 0.7}$$

$$\leq \text{Field-N} \times 2$$

where, p_o = Effective overburden pressure in kg/cm^2 $< 2.8\ kg/cm^2$

The field N-values have been corrected for overburden using the above expression. The field as well as corrected N-values are presented in Table 3.9.

TABLE 3.9

FIELD AND CORRECTED N-VALUES

Bore hole No.	1		2		3	
Depth of WT on the test day m	0.70		2.0		2.65	
Depth m	Field N	Corrected N	Field N	Corrected N	Field N	Corrected N
1.0	9	18	4	8	5	10
1.5	8	16	4	8	4	8
2.0	6	12	4	8	5	10
2.5	10	20	9	18	6	12
3.0	7	14	6	12	7	14
3.5	5	10	6	12	5	10
4.0	5	10	10	20	2	4

The standard penetration test results are to be used only as the guidelines for arriving at a proper bearing capacity value for clayey soils. Many charts are available in literature, for example, Bazara's chart, Peck's chart, Mayerhof's chart, Terzaghi and Peck's chart, which give the allowable bearing capacity for sandy soils for a settlement of one inch based on N-values. One such method is described here.

Terzaghi and Peck have given the following expression for allowable bearing capacity for 1" settlement.

$$q_{all} = 720\ (N-3)\ \left(\frac{B+1}{2B}\right)^2\ K_d\ C_w$$

where, q_{all} = Allowable bearing capacity in pounds/ft^2

N = Corrected N-value

B = Width of foundation in ft.

K_d = Depth factor $= 1 + \frac{D}{B} \leq 2.0$

C_w = Correction factor for ground water table (WT).

= 0.5 if WT is at the base of the foundation

= 1.0 if WT is at a depth B below the foundation level.

The above expression can be written in the following

form.

$$q_{all} = 0.352\ (N-3)\ \left(\frac{B+30}{2B}\right)^2\ K_d\ C_w$$

where q_{all} is in kg/cm^2 and B in cms.

Using this expression, the bearing capacity for a foundation of 1m width at a depth of 1m from ground surface has been calculated and is presented in Table 3.10. The value of C_w has been taken as 1.0 for bore hole 1 test data as the ground water level was above the foundation level when the test was conducted, and as 0.5 for bore hole 2 and 3 test data since the ground water level was at a depth equal to or more than B (=1.0m) below the foundation level (D=1.0m) on these test days, and the water table is known to rise upto and above the foundation level.

TABLE 3.10

ALLOWABLE BEARING CAPACITY BASED ON N-VALUES

Bore hole	C_w	K_d	N	q_{all} kg/cm^2
1	1.0	2	18	4.46
2	0.5	2	8	0.74
3	0.5	2	10	1.04

These values of allowable bearing capacity match well with those obtained by Jain (1977) based on his standard penetration tests.

CHAPTER 4

CORRELATION AND EVALUATION OF DESIGN PARAMETERS FOR SHALLOW FOUNDATIONS

4.1 General :

The results from plate load tests in terms of allowable bearing capacity and deformation modulii are examined to assess the effect of plate size and natural moisture content. The standard penetration test data have been correlated with cone resistance, undrained cohesion, drained elastic modulus and allowable bearing capacity, and the correlation factors thus obtained are compared with those available in literature for similar soils.

4.2 Bearing Capacity :

4.2.1 Effect of In-situ Moisture Content :

The allowable bearing capacity values have been plotted against the natural moisture content values in Fig. 4.1(d). The plot clearly indicates two zones: one at smaller moisture contents and the other at higher moisture contents. Jain (1977) and Srivastava (1977)'s test results have also been plotted in this figure which fall in their respective zones corresponding to their

moisture content values. The allowable bearing capacity values as obtained from the tests conducted on the relatively dry soil group themselves in one zone as against those obtained from tests on relatively wet soils (nearly saturated soils) which fall in another zone. Fig. 4.1(d) indicates that the bearing capacity decreases as the soil gets saturated, even though the limited data given here does not allow to suggest an exact relationship relating the bearing capacity with the moisture content of the soil. More data points in Fig. 4.1(d) would be highly desirable to define such a relationship.

4.2.2 Effect of Plate Size :

The allowable bearing capacity values for a factor of safety of 3 have been plotted against the width of the test plates in Fig. 4.1(b). Dr. Yudhbir has provided the test results of the plate load test conducted at Panki Thermal Power Plant near IIT Kanpur Campus with a test plate of 2m x 2m size which is also shown on this plot. The results from Jain (1977)'s plate load tests are also shown in this figure. The plot indicates that the bearing capacity decreases upto a width of 60 cm beyond which it remains constant. This agrees very well with the ISI recommendation that the bearing capacity as

obtained from a plate load test with a 60 cm plate should be taken as the bearing capacity value for the actual foundation on silty and clayey soils and on loose to medium compact sandy soils (N <15).

4.2.3 Recommended Safe Allowable Bearing Capacity :

From the discussion on the effect of the plate size on bearing capacity and the ISI recommendation, it would appear that the allowable bearing capacity (9.2 t/m^2, Table 3.7(a)) for the 60 cm plate test conducted with water level coinciding with the plate level (foundation level also) should be adopted for design for the worst condition in these soils. However, this would result in a predicted settlement of the footing (B=1m at D=1m) as 6 mm only on the basis of load settlement data obtained. Here , according to the ISI recommendation the settlement for, the footing width has been increased in direct proportion to the size of the footing as compared to that of the plate. Incidently, this ISI recommendation is substantiated by the Panki plate load test for a 2m x 2m plate. From the load-settlement data of Panki test, the settlement under a stress of 9.2 t/m^2 was about 14 mm which agrees well with the value of 12 mm as obtained by adjusting in direct proportion the results of 60 cm plate test.

I I T. KANPUR
CENTRAL LIBRARY
Acc. No. A 54908

Now if all the 6 mm under 9.2 t/m^2 is assumed as initial settlement only (a conserviative assumption) and if the ratio between initial and total settlement for these soils is taken as 40 % (a conservative value, it varies from 40-70 % , as shown later) the total expected settlement of the footing will be 15 mm only compared to 40 mm as now recommended by ISI and also 25 mm as commonly used in practice for these soils. Clearly the value of 9.2 t/m^2 as allowable bearing capacity is conservative and leaves scope for improvement in the design of foundations.

It is recommended that instead of a factor of safety of3 as used to obtain 9.2 t/m^2/, it may be lowered to a value of 2. The corresponding allowable bearing capacity will then be 13.8 t/m^2 with an initial settlement of 11.7 mm and a total settlement of 29.2 mm. These values are within the allowable settlements and as such safe. It is thus recommended that allowable bearing capacity of about 14 t/m^2 and net allowable bearing capacity of 12 t/m^2 (for computation of settlement) should be adopted in the design of shallow foundation under similar conditions. These values agree well with the value of 12-15 t/m^2 as obtained from 2 q_c, where q_c is the cone resistance in t/m^2. For these conditions, $q_c \approx 1.5$ N and N varies between 4 and 5 . From total stress and effective stress analysis on the basis of

tests on saturated block samples, Srivastava (1977) recommends q_{all} between 15-20 t/m^2. All these previous results seem to justify the recommendation made here.

4.3 Deformation Modulus :

The undrained secant modulus has been plotted against the natural moisture content in Fig. 4.1(c). This plot exibits clearly two different zone at higher and lower moisture contents as in the plot of q_{all} vs W_n. Jain (1977) and Srivastava (1977)'s points also lie in these zones. The plot clearly indicates that the deformation modulus decreases as the moisture content (i.e. the degree of saturation) increases even though the limited data presented here does not allow to suggest an exact relationship relating the deformation modulus with the moisture content of the soil.

Fig. 4.1(a) shows a plot of Δ vs q_{all} B. The slope of this plot gives the value of $\frac{1-\nu^2}{E} \frac{\pi}{4}$. The values of deformation modulus for saturated and partially saturated soil conditions have been calculated as shown in the figure. These values are relatively higher than the secant modulus obtained earlier, which is expected, as the secant modulus has been found out at the stresses equal to half of the ultimate stresses while these values

correspond to a stress level equal to one third of the ultimate stresses (q_{all} used here is for a factor of safety equal to 3).

4.4 Coefficient of Volume Compressibility :

The theory of elasticity provides the following relationship for drained secant modulus in terms of coefficient of volume compressibility (m_v).

$$E' = \frac{(1+\nu')(1-2\nu')}{m_v(1-\nu')}$$

For $\nu' = 0.28$ (Srivastava, 1977)

$$E' = \frac{0.782}{m_v}$$

The coefficient of volume compressibility has been found out using the above relationship for the drained secant modulus (602 t/m^2, Table 3.8) as obtained from the plate load test conducted on saturated soil with a 60 cm plate, and is equal to 0.0013 m^2/t.

The value of m_v obtained by Srivastava (1971) for these soils is 0.002 m^2/t and keeping in mind that field settlement of structures is usually less than that based on laboratory tests, the value of 0.0013 m^2/t estimated here appears to be more realistic one and is recommended for the estimation of settlement of shallow foundations on such soils.

4.5 Settlement Analysis :

The initial (immediate) and the total settlements have been calculated for a footing size of 20m x 1m at 1m depth using different methods for allowable bearing capacity of 14 t/m^2 (net allowable bearing capacity equal to 12 t/m^2).

4.5.1 Elastic Method :

Srivastava (1977) has estimated a value of K_o (coefficient of lateral stress at rest) equal to 1.0 for these soils which gives a value of zero for initial stress ratio ($f = \frac{(1-K_o)\sigma_v'}{S_u}$). This yields a value of 1.7 (D'Appolonia et.al. 1971) for the factor of safety at first local yield which is less than 2, the value used here, so there will be no initial yielding during undrained loading in these soils.

Initial settlement (S_i) is calculated using the relationship given by Bowles (1968).

$$S_i = \mu_0 \mu_1 q B \frac{1-\nu^2}{E_u}$$

Where, μ_0 and μ_1 depend on the shape and location of the foundation below the ground level and depth of strata (H) below the foundation level.

For, $\frac{L}{B} = 20$, $\frac{H}{B} = 4.0$ and $\frac{D}{B} = 1.0$

$\mu_0 = 0.9$ and $\mu_1 = 1.20$

$$S_i = 1.2 \times 0.9 \times 12 \times 100 \times \frac{1 - (0.50)^2}{708}$$

$$= 1.37 \text{ cm}$$

And, total elastic settlement $= \mu_0 \mu_1 q B \frac{1-\nu'^2}{E'}$

$$S_t = 1.2 \times 0.9 \times 12 \times 100 \times \frac{1-(0.28)^2}{602}$$

$$= 1.98 \text{ cm}$$

So,

$$\frac{S_i}{S_t} = \frac{1.37}{1.98} = 0.7$$

here, initial settlement has been computed using undrained secant modulus (708 t/m^2) which is very low compared to the initial modulus (1448 t/m^2, Table 3.8).

4.5.2 1-D Compression Method :

This method provides the following expression for one dimensional settlement (S_{1-D}).

$$S_{1-D} = m_v (\Delta P) H_1$$

Where, P = Increase in vertical stress

H_1 = Thickness of the zone of influence

m_v = Coefficient of volume compressibility.

S_{1-D} = 0.0013 x (4 x C.137 x 12) x 200

= 1.71 cm

S_t = $S_i + \mu_o S_{1-D}$ (Poulos, 1975)

= 1.37 + 0.9 x 1.71 = 2.91 cm

$$\frac{S_i}{S_t} = \frac{1.37}{2.91} = 0.47$$

4.5.3 Skempton and Bjerrum's Method :

Skempton and Bjerrum (1957) have suggested that the final settlement can be calculated using the following expression

$$S_t = S_i + \mu\mu_o S_{1-D}$$

Where, μ depends on the pore pressure parameter (A) and the depth of strata (H) below foundation level.

For, A = 0.4 (Srivastava, 1977), and $\frac{H}{B} = 4$

$\mu = 0.6$

S_t = 1.37 + 0.6 x 0.9 x 1.71 = 2.29 cm

$$\frac{S_i}{S_t} = \frac{1.37}{2.29} = 0.60$$

4.5.4 Method Based on $\frac{1}{m_v S_u}$ Value:

Srivastava (1977) has recommended a value of S_u = 9 t/m^2 for these soils which gives $\frac{1}{m_v S_u}$ equal to 85.

The value of $\frac{1}{m_v\ S_u}$ equal to 85 agrees well with that obtained from the relationship developed by Skempton (1951), for $\frac{S_t}{B}$ = 1.5 % and 2.9 % for factor of safety of 3 and 2 respectively as estimated from plate load test.

The total settlement is found from the following expression (Skempton, 1951).

$$\frac{S_t}{B} = \frac{5}{1/m_v S_u} \times \frac{1}{F}$$

Where, F is factor of safety

For F=2 , $\frac{S_t}{B} = \frac{5}{85 \times 2} = 0.029$, $S_t = 2.9$ cm

For F=3 , $\frac{S_t}{B} = \frac{5}{85 \times 3} = 0.019$, $S_t = 1.9$ cm

It should be noted that the total settlement as obtained by 1-D compression method (2.91 cm) and by the method based on $\frac{1}{m_v S_u}$ value (S_t = 2.9 cm) matches exactly with that arrived at earlier (29.2 mm) on the basis of 60 cm plate load test data.

It is interesting to note that the ratio of initial to total settlement, $\frac{S_1}{S_t}$ = 0.7 is for an ideal homogeneous isotropic and linearly elastic material.

From $S_1 = \mu_0 \mu_1\ q\ B\ \frac{1-\nu^2}{E_u}$ and ,

$$S_t = \mu_0 \mu_1 \, q \, B \, \frac{1-\nu'^2}{E'} ,$$

$$\frac{S_i}{S_t} = \frac{E'}{E_u} \times \frac{1-\nu^2}{1-\nu'^2} \qquad (4.1)$$

Substituting $E_u = \dfrac{3E'}{2(1+\nu')}$ in equation 4.1

$\dfrac{S_i}{S_t} = \dfrac{1}{2(1-\nu')}$ is obtained for $\nu = 0.5$

clearly this ratio depends on ν' only.

With $\nu' = 0.28$ for these soils, $\dfrac{S_i}{S_t} = 0.7$

However Hooper (1975) has shown that for homogeneous cross anisotropic elastic soil, the ratio S_i/S_t depends on two anisotropic parameters, $m' = \dfrac{G_{vh}}{E_v}$ and $n' = \dfrac{E'_h}{E'_v}$.

Srivastava (1977) has shown that $\dfrac{E'_h}{E'_v} = 1.0$ for these soils in the dessicated zone. With $n' = 1.0$, Hooper (1975) shows that $\dfrac{S_i}{S_t}$ will vary between 0.4-0.7 for $m' = 1.0$ to 0.33 respectively. This brings out an important point that for anisotropic natural sediments, $\dfrac{S_i}{S_t}$ will decrease from its high value for isotropic materials. The values 0.47-0.70 computed here appear to be reasonable and an assumption of $\dfrac{S_i}{S_t} = 0.4 - 0.5$ appears quite applicable for these soils.

Elastic method clearly gives the upper range for this ratio.

4.6 Correlation Between N-Value and Cone Resistance :

The cone resistance (q_c) values have been taken from static cone penetrometer test data of Jain (1977) to correlate the N-values with the q_c-values.

The ratio (n) of cone resistance (q_c) to field N-value is found to vary between 1 and 2 which agrees well with the Jain (1977)'s finding and it compares well with n=2, recommended by Meyerhof (1965) for silts, sandy silts and slightly cohesive silt sand mixtures based on test data from many countries.

4.7 Correlation between Cone Resistance and Undrained Cohesion :

The cone resistance (q_c) values have been taken from Jain (1977) ,and a value of 0.9 kg/cm^2 for undrained cohesion (S_u) as found by Srivastava (1977) for these soils has been used to find the correlation factor between the cone resistance and undrained cohesion.

The ratio (β) of cone resistance (q_c) to undrained cohesion (S_u) is found to vary between 6 and 14 for these soils. Sanglerat (1972) has observed that this ratio ($\beta = q_c/S_u$) lies between 10 and 20 for soft clays of Anney area (France). The values obtained for IIT Kanpur Campus soil are little ~~higher~~ lower than those observed by Sanglerat since

Campus soil is stiffer and lightly overconsolidated.

4.8 Correlation between Cone Resistance and Drained Elastic Modulus :

Jain (1977)'s static cone penetrometer test data have been used to find the correlation between cone resistance and drained elastic modulus.

The ratio (α) of drained elastic modulus (E') to cone resistance (q_c) is found to vary between 4 and 10 which agrees well with the Sanglerat (1972)'s findings ($3.5 < \alpha < 13$) for overconsolidated clays (for normally consolidated clays, $1.7 < \alpha < 5.4$), and with Jain's findings as well.

4.9 Correlation between Cone Resistance and Allowable Bearing Capacity:

The static cone penetrometer test data of Jain (1977) have been used to correlate the cone resistance (q_c) with the allowable bearing capacity.

The ratio (m) of cone resistance (q_c) to allowable bearing capacity is found to vary between 4 and 10. Sanglerat (1972) has recommended a value of 10 for this ratio ($m=q_c/q_{all}$) for shallow foundations on sands, stiff clays, sandy clays and silty sands at 1m depth, which gives a very conservative

estimate of allowable bearing capacity so a value of 5 for m is usually recommended which seems to agree well with the observations on I.I.T. Kanpur Campus soil.

CHAPTER 5

CONCLUSIONS

The following are the conclusions based on the soil investigations made in south-western part of the I.I.T. Kanpur Campus.

1. The ground water table plays a very significant role in altering the soil properties. There can be a tremendous difference in soil parameters as obtained from different tests depending upon the location of the ground water table on the day of testing. The water table is found to vary enormously (with reference to the shallow foundations normally located at a depth of 1 m) in this area as shown in Fig. 2.2. This alternate rise and fall in ground water level causes softening and stiffening respectively in the soil which leads to a dessicated zone in the shallow depths and the soil of this zone behaves as a lightly overconsolidated soil and not as a normally consolidated soil as is usually expected of the alluvial soils.

2. A representative bore hole profile for south western part of I.I.T. Kanpur Campus has been drawn

(Fig. 5.1) based on all the test data available for this area. The profile clearly demarcates the zone of dessication at 3m below ground surface at which level, the soil properties change significantly.

3. The allowable bearing capacity values are found to decrease with increase in plate size upto 60 cm. plate beyond which these remain constant. (Fig. 4.1 (b)). ISI also recommends the use of 60 cm plate size for conducting the plate load test to determine the allowable bearing capacity of actual size foundation without making any further correction for the size of the test plate.

4. The settlement at allowable stress as obtained from a 60 cm plate load test can be proportionately increased for the size of footing to get the corresponding settlement of the actual size footing. Assuming all of it to be initial settlement and the ratio of initial settlement to final settlement equal to 0.40, the final settlement of the footing can be calculated. It has been shown that the final settlement estimated from plate load test conducted with water level coinciding with foundation level agrees well with the values obtained from other theoretical and semi-emperical methods.

5. The allowable bearing capacity values decrease as the ground water level comes up (Table 3.6 with Table 3.1).

6. The allowable bearing capacity and deformation modulii decrease as the moisture content increases (Fig. 4.1 (c) & (d)) i.e. as the degree of saturation of the soil increases.

7. All the tests whether they be field or laboratory tests for determining the soil parameters, should be performed in saturated soil condition, and if the tests have been carried out in partially saturated or dry soil conditions, the soil parameters thus obtained should be suitably corrected for the moisture content so as to arrive at correct soil parameters for saturated soil condition (Fig. 4.1 (c) & (d)).

8. ISI method of interpreting the load-settlement data (load vs. settlement on a log-log plot) yields a conservative estimate of allowable bearing capacity for a factor of safety equal to 2. This is clear from the fact that the settlements even at ultimate stresses are much smaller compared to the usually allowable settlements (25-40 mm) for buildings on such soils. Kee's method is recommended for use to interpret the plate load test data.

9. The following soil parameters are recommended for the use in the design of shallow foundations on such soils as encountered in south-western part of I.I.T. Kanpur Campus.

(i) Allowable bearing capacity (q_{all}) = 14 t/m^2

(ii) Initial deformation modulus (E_i) = 1448 t/m^2

(iii) Undrained secant modulus (E_u) = 708 t/m^2

(iv) Drained secant modulus (E') = 602 t/m^2

(v) Undrained cohesion (S_u) = 9 t/m^2

(vi) Coefficient of volume compressibility (m_v) = 0.0013 m^2/t

(vii) $\frac{1}{m_v S_u} = 85$

(viii) Strength parameters (Srivastava, 1977)

(a) Total stress parameters.

(1) Unconsolidated undrained

$c = S_u = 9.0\ t/m^2, \quad \phi = 0$

(2) Consolidated undrained

$c = 4.8\ t/m^2, \quad \phi = 17.5°$

(b) Effective stress parameters:

$c' = 3.9\ t/m^2, \quad \phi' = 25°$

Pore pressure parameter, A = 0.4 for F = 2-3

BIBLIOGRAPHY

1. Bowles, J.E., 1968, "Foundation Analysis and Design," MgGraw Hill Book Company, N.Y.

2. D' Appolonia, D.J., Poulos, H.G., and Ladd, C.C., 1971, "Initial Settlement of Structures on Clay," Proc. ASCE, Vol. 97, No. SM 10.

3. Jain, C.K., 1977, "Evaluation of In-Situ Parameters for Design of Shallow Foundation On Alluvial Deposits," M.Tech. Thesis, I.I.T. Kanpur.

4. Srivastava, N.K., 1977, "Laboratory Determination of In-Situ Parameters of Alluvial Soils," M.Tech. Thesis, I.I.T. Kanpur.

5. Pec', R.B., Hensen, W.F., and Thornburn, T.H., 1973, "Foundation Engineering," John Wiley & Sons, New York.

6. Skempton, A.W., 1951, "The Bearing Capacity of Clays," Building Research Congress, England.

7. Skempton, A.W., and Bjerrum, L., 1957, "A Contribution to the Settlement Analysis of Foundation on Clay," Geotechnique, Vol. 7.

8. Winterborn, H.F., and Fang, H.Y., 1975, "Foundation Engineering Handbook," Van Nostrand Reinhold Company, New York.

9. Kee, C.F., 1972, "The Inverse Slope as a Prediction of Ultimate Bearing Capacity of Piles," Proc. Third S.E.A. Conf. on Soil Engineering, Hong Kong.

10. Lambe, T.W., and Whitman, R.V., 1973, "Soil Mechanics," Wiley Eastern Ltd., N.Y.

11. Meyerhoff, G.G., 1965, "Shallow Foundations," J.S.M.F.E. Div., ASCE, Vol. 91, No. SM2.

12. Sanglerat, G., 1972, "The Penetrometer and Soil Exploration," Elsevier Publishing Co., Amsterdam.

13. Terzaghi, K., Peck, R.B., 1967, "Soil Mechanics in Engineering Practice," John Wiley & Sons, N.Y.

14. Holtz, W.G., and Hilf, J.W., 1961, "Settlement of Soil Foundations due to Saturation," Proc. 5th Int. Soc. S.M.F.E. (Paris), Vol. 1.

15. Konder, R.L., 1963, "Hyperbolic Stress-strain Response : Cohesive Soils," J.S.M.F.E. Div., ASCE, Vol. 89, No. SM1.

16. Meyerhof, G.G., 1951,"The Ultimate Bearing Capacity of Foundations," Geotechnique, Vol. 2.

17. Meyerhof, G.G., 1965, "Shallow Foundations," Proc. ASCE, Vol. 11, No. SM2.

18. Poulos, H.G., 1975,"Settlement of Isolated Foundations," Research Report No. R265, School of Civil Engineering, University of Sydney.

19. Hooper, J.A., 1975, "Elastic Settlement of a Circular Raft in Adhesive Contact With a Transversely Isotropic Material," Geotechnique, Vol. 25, No. 4.

20. Barden, L., 1963, "Stresses and Displacement in a Cross-anisotropic Soil," Geotechnique, Vol. 13.

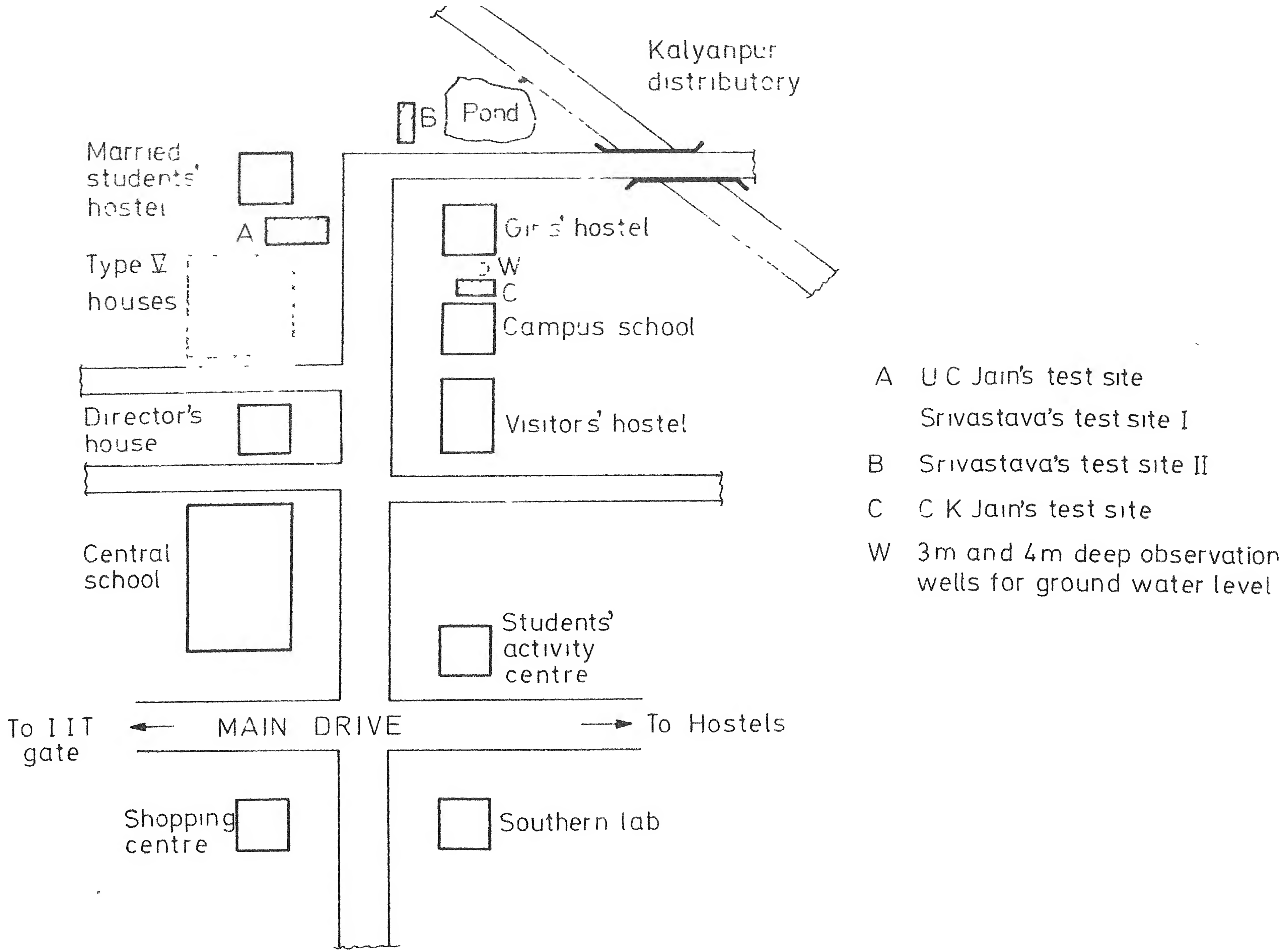


FIG 21 LOCATION OF TEST SITE IN CAMPUS (Not to scale)

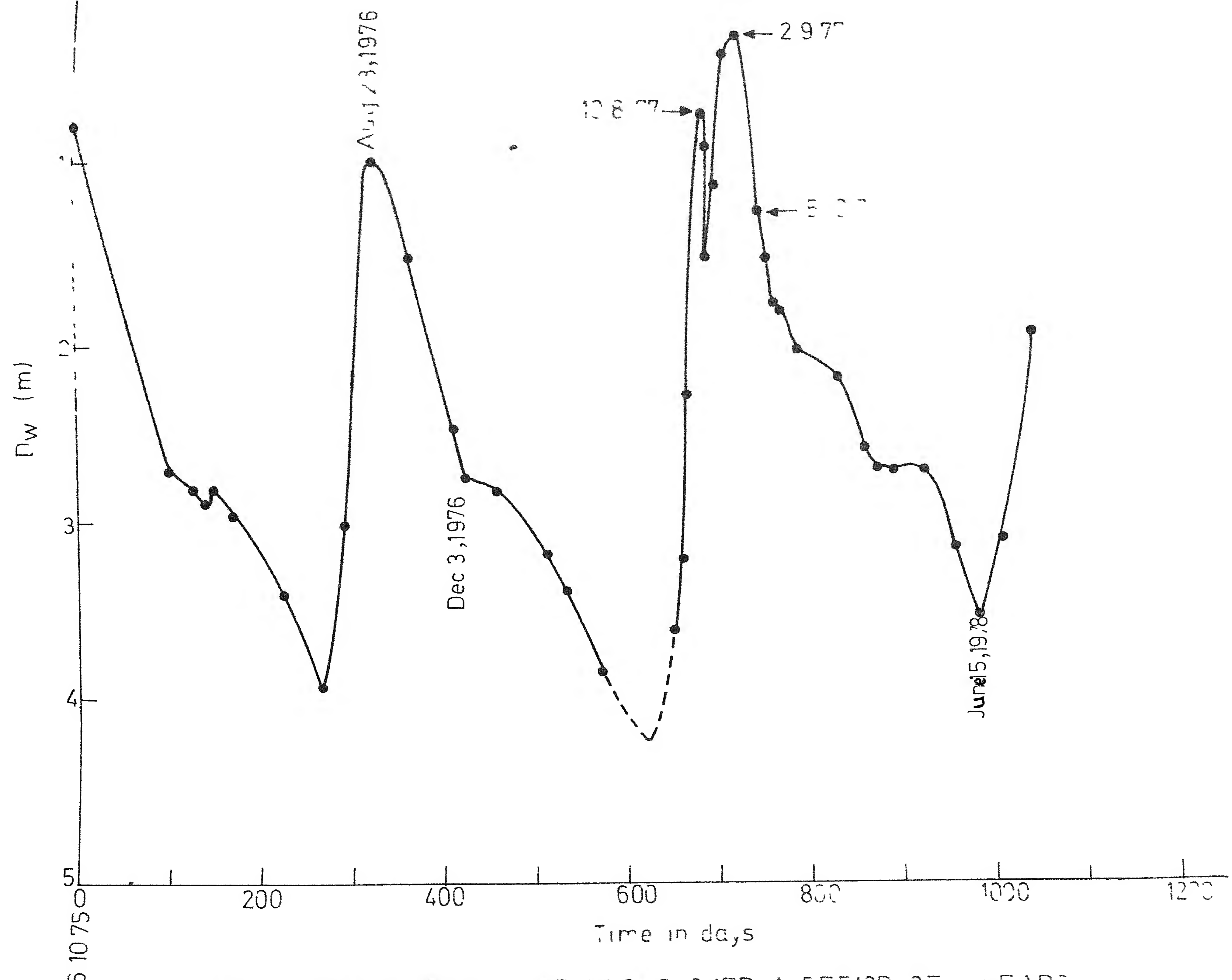


FIG 2.2 WATER TABLE VARIATIONS OVER A PERIOD OF YEARS

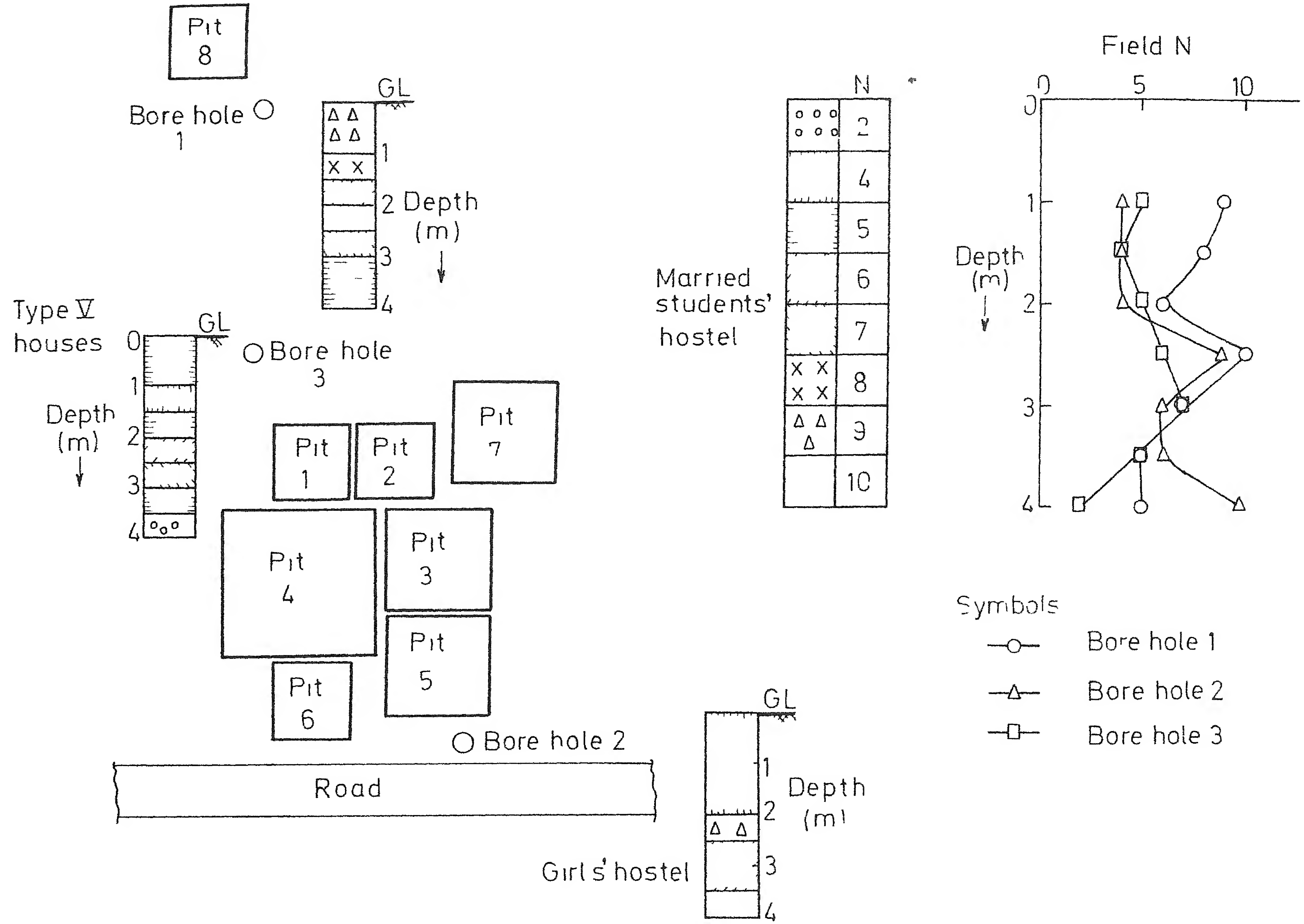


FIG 2 3 LOCATION OF TEST PITS AND BORE HOLES AT TEST SITE (Not to scale)

PLASTICITY CHART

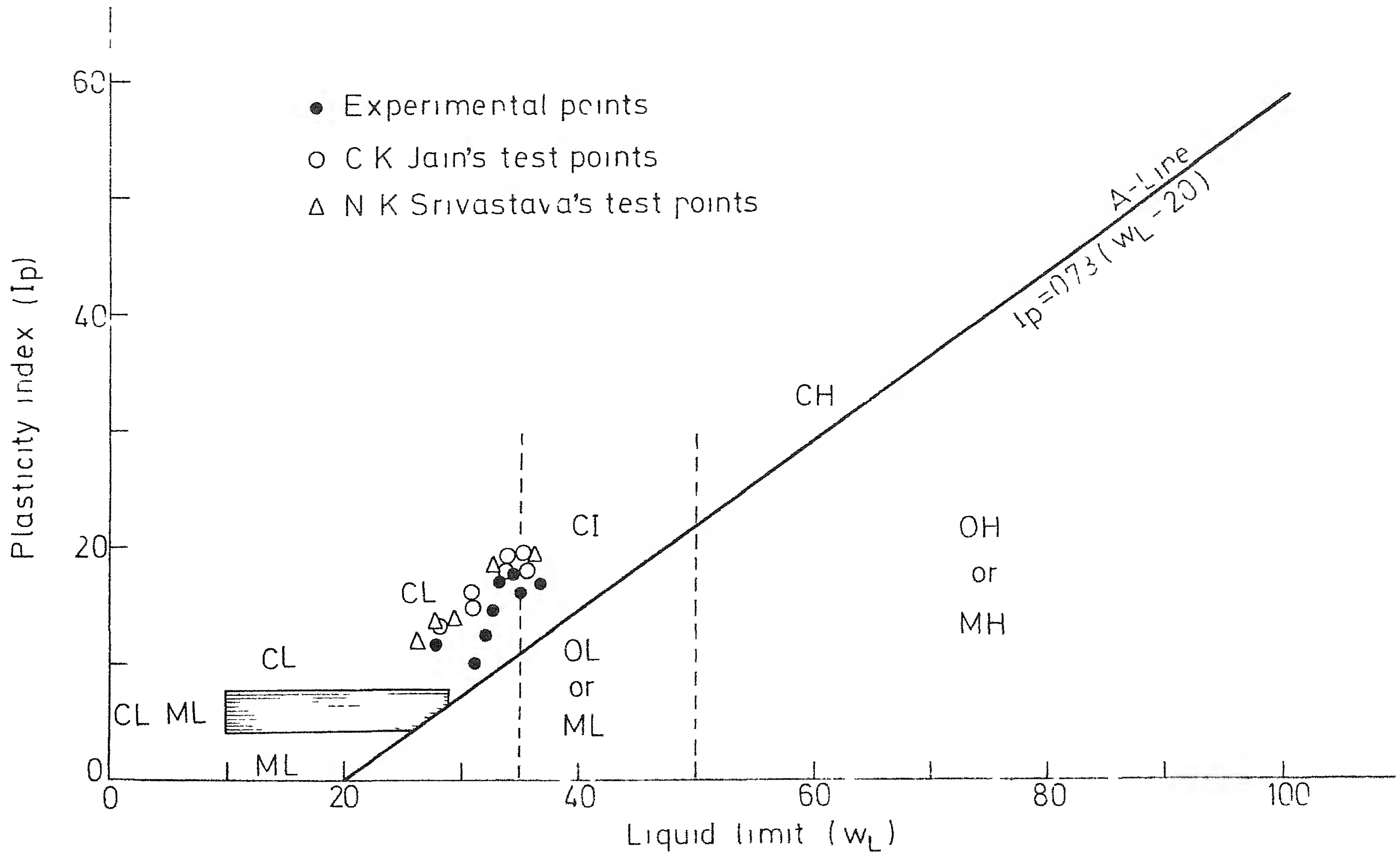
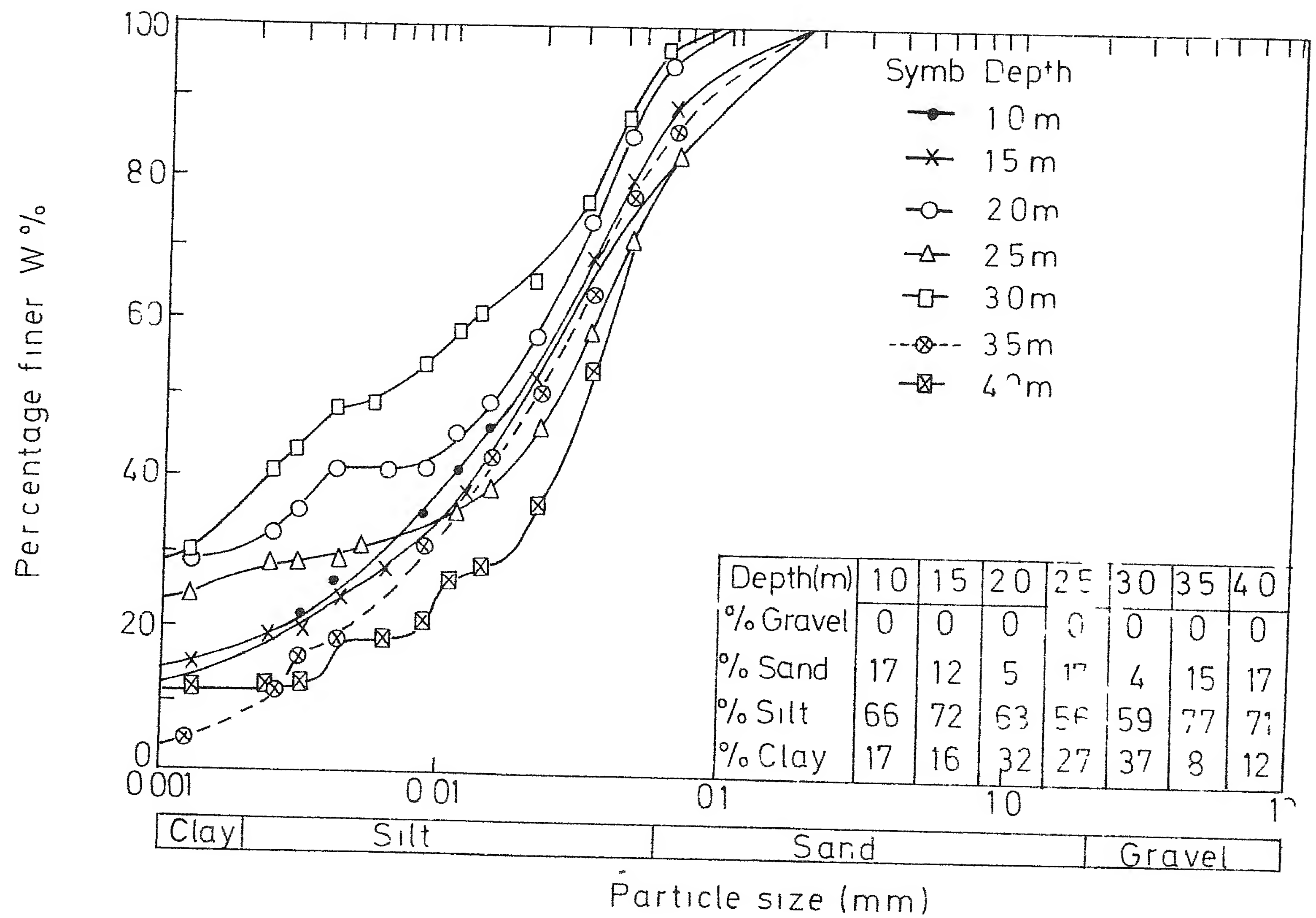
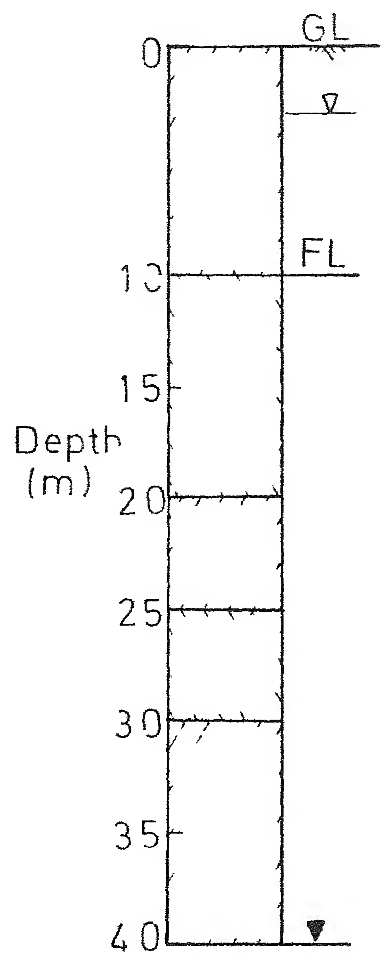


FIG 2.4 CLASSIFICATION OF SOILS



Depth(m)	10	15	20	25	30	35	40
% Gravel	0	0	0	0	0	0	0
% Sand	17	12	5	17	4	15	17
% Silt	66	72	63	56	59	77	71
% Clay	17	16	32	27	37	8	12

FIG 2 5 PARTICLE SIZE DISTRIBUTION CURVE



Description of strata	Soil classi-fication	W_P %	W_L %	[illegible]	Fine sand %	Silt %	Clay %
Clayey Silt with fine Sand	CL	17.8	33.5	[illegible]	[illegible]	66	17
Clayey Silt with fine Sand	CI	17.8	34.5	16.8	12	72	16
Clayey Silt	CI	19.5	34.8	18.6	5	63	32
Clayey Silt with fine Sand	CL	19.7	32.5	17.8	17	56	27
Clayey Silt	CI	20.2	[illegible]	19.8	4	59	37
Sandy Silt with Clay	CL	26.6	32	[illegible]	15	77	8
Sandy Silt with Clay	CL	22.6	31	[illegible]	17	71	[illegible]

▽ Highest ground water level

FL Usual level for shallow foundations

▼ Lowest ground water level

FIG 26 A BORE HOLE PROFILE

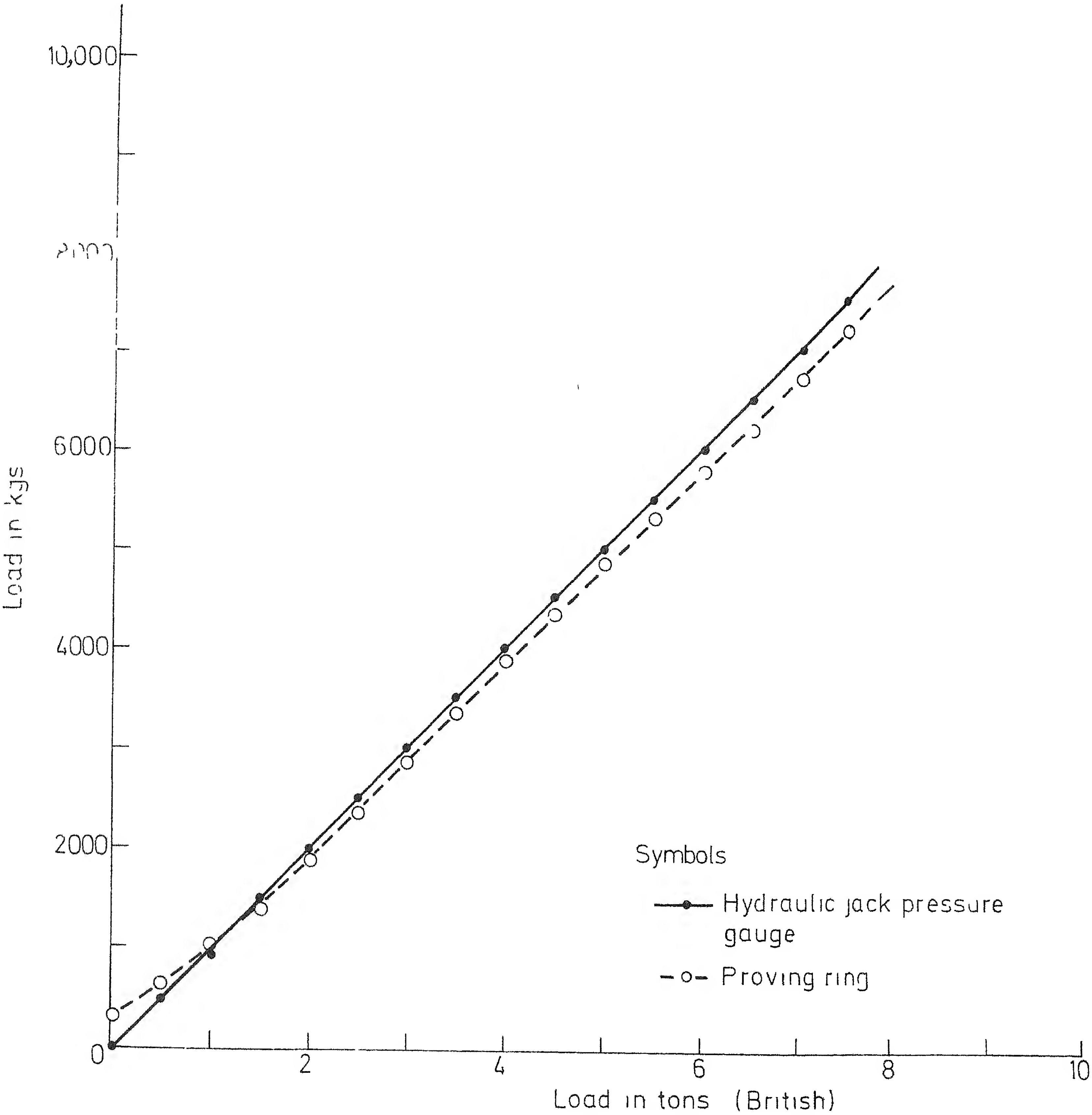


FIG 31 JACK CALIBRATION

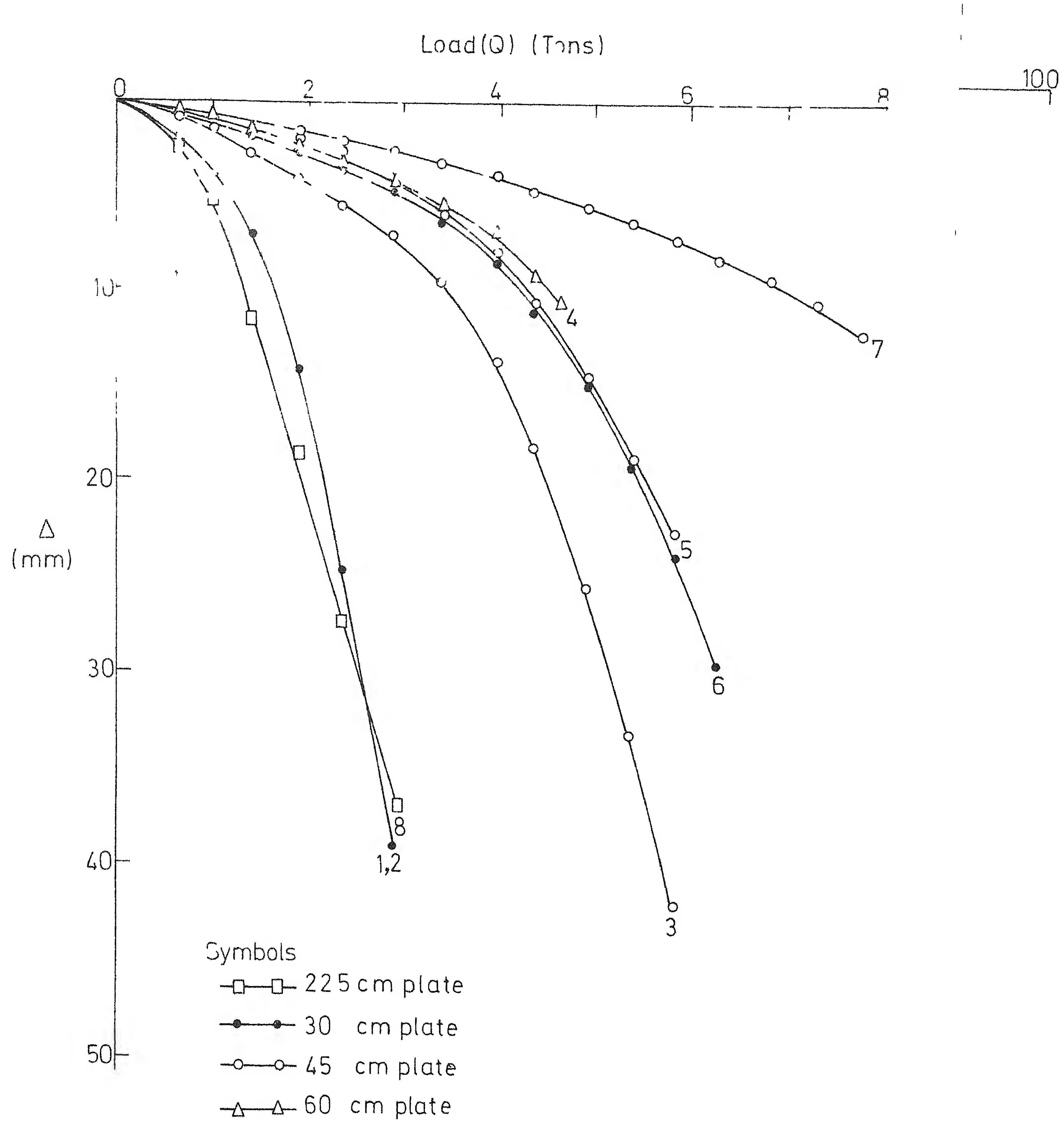


FIG 32 LOAD VS SETTLEMENT PLOT

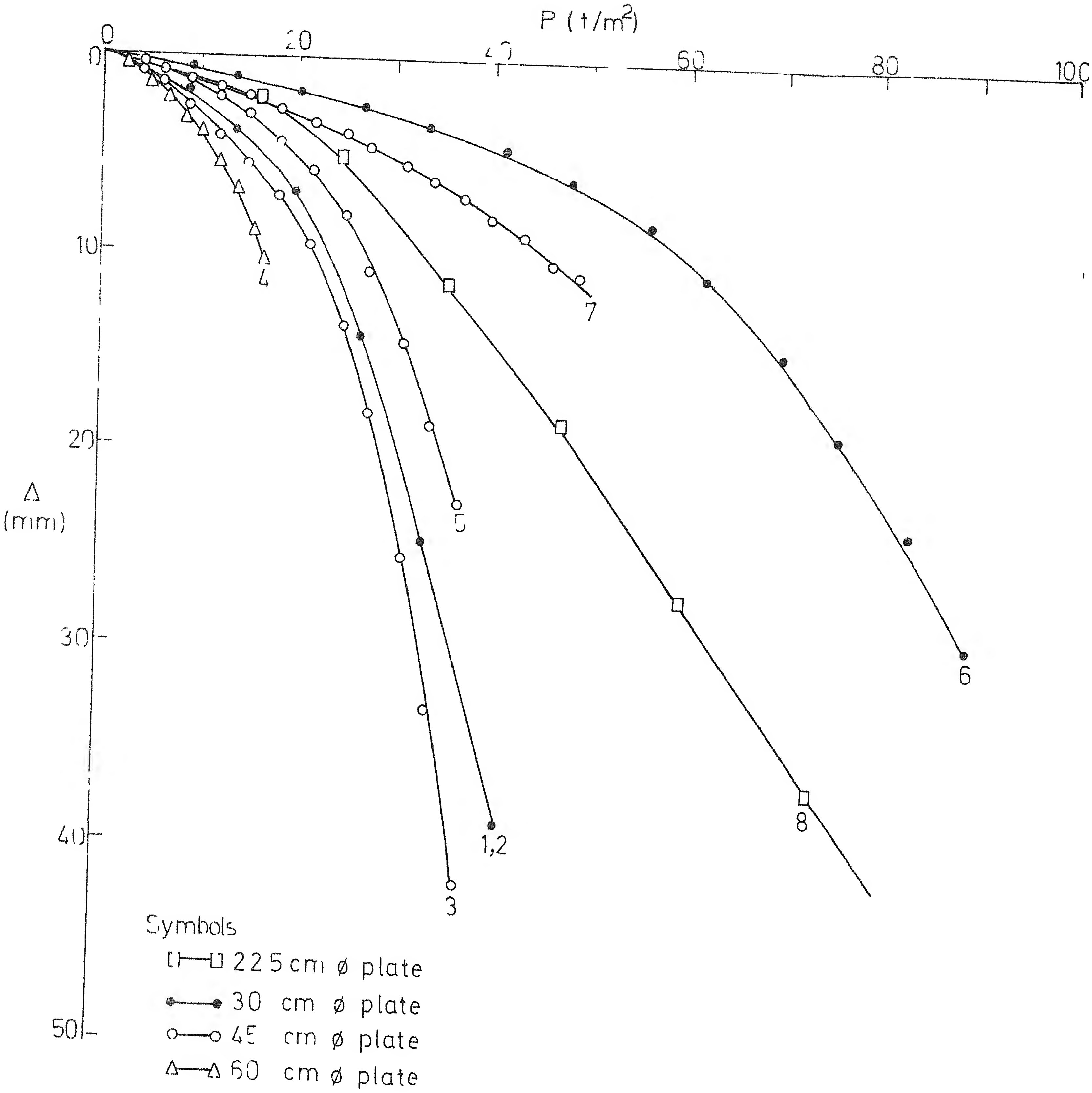


FIG 3.3 STRESS (Load / Area) VS SETTLEMENT PLOT

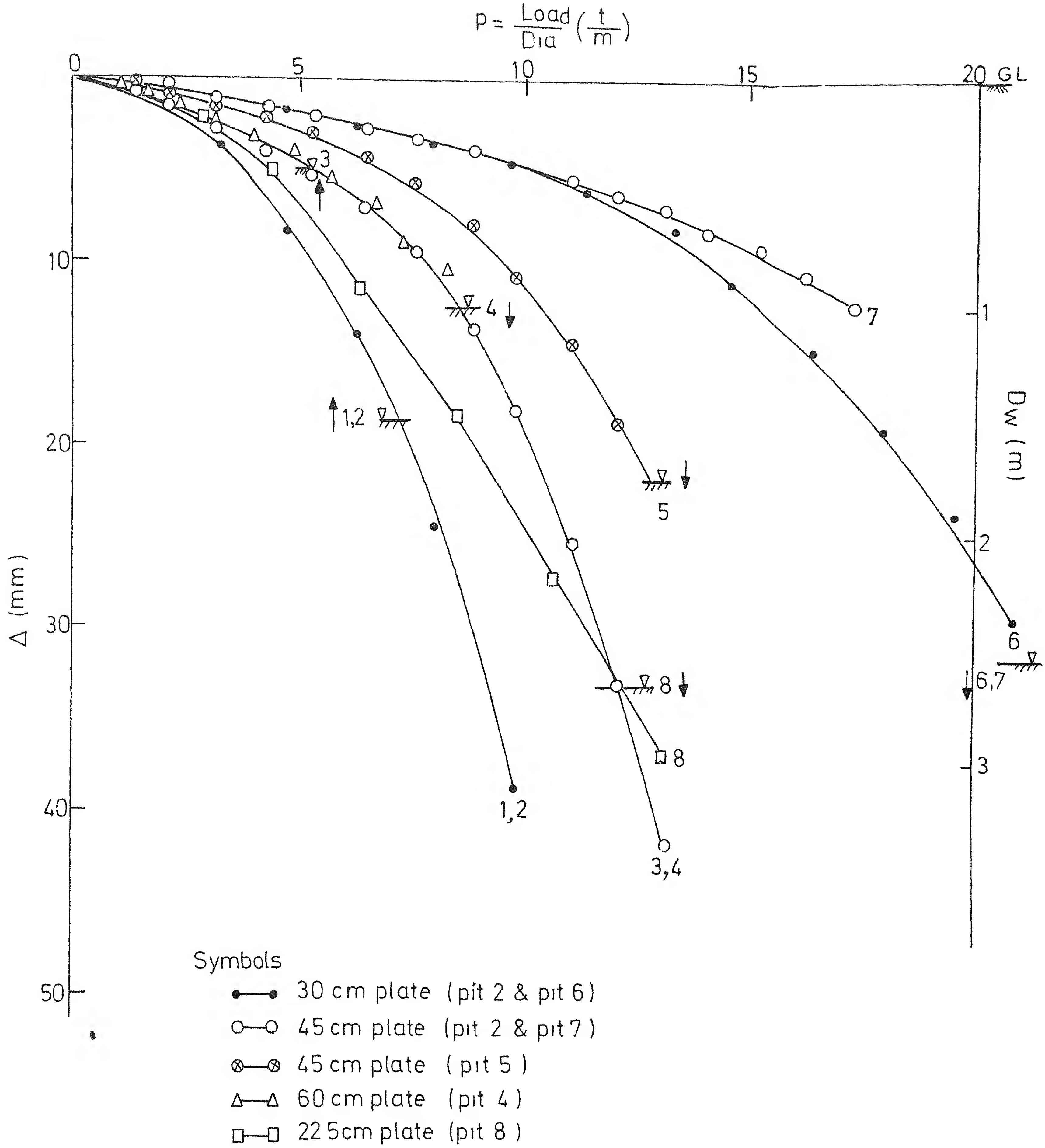


FIG 3 4 NORMALIZED LOAD (LOAD/DIAMETER) VS SETTLEMENT PLOT

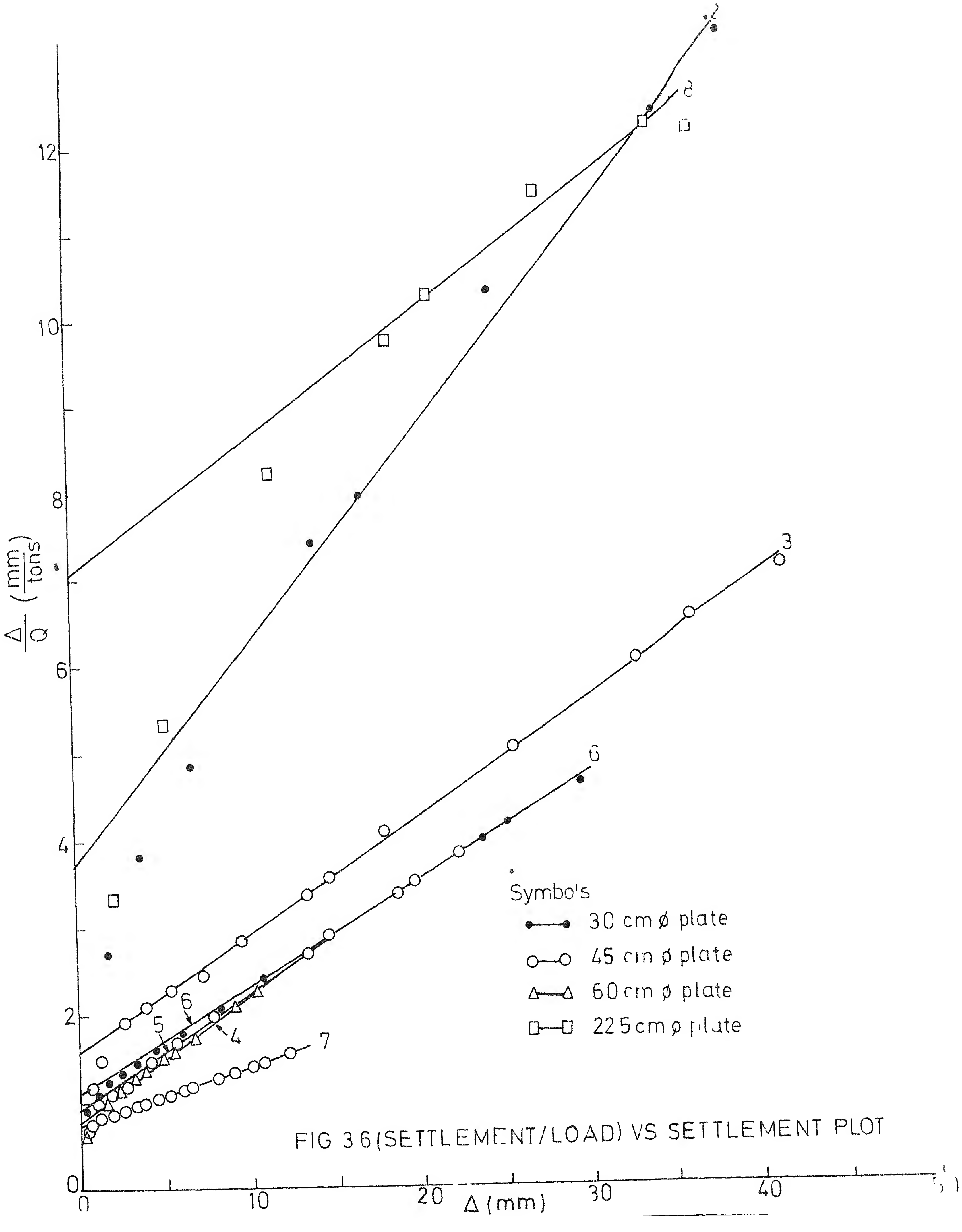


FIG 36 (SETTLEMENT/LOAD) VS SETTLEMENT PLOT

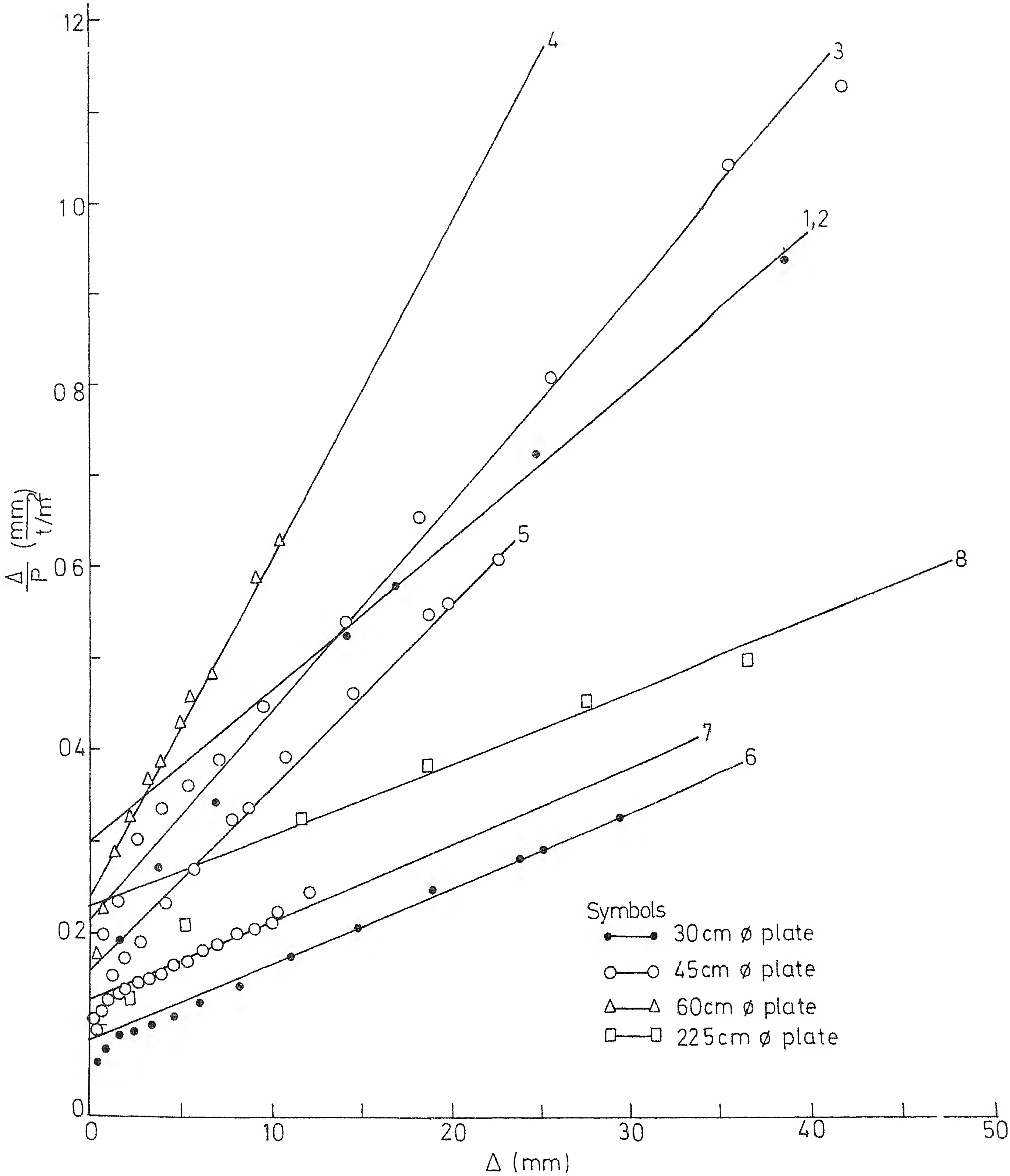


FIG 37 (SETTLEMENT/ STRESS) VS SETTLEMENT PLOT

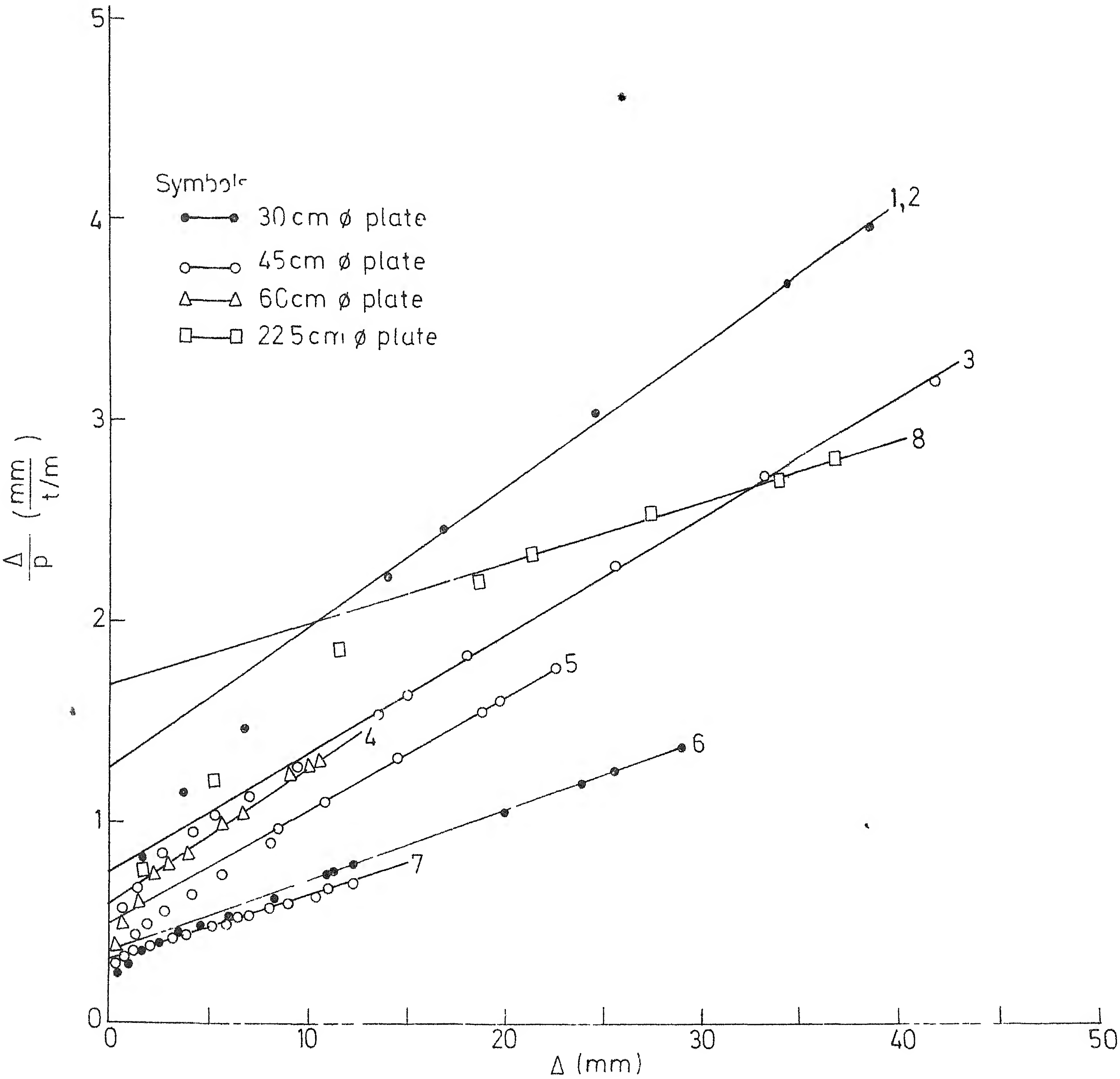


FIG 38 (SETTLEMENT/NORMALIZED LOAD) VS SETTLEMENT

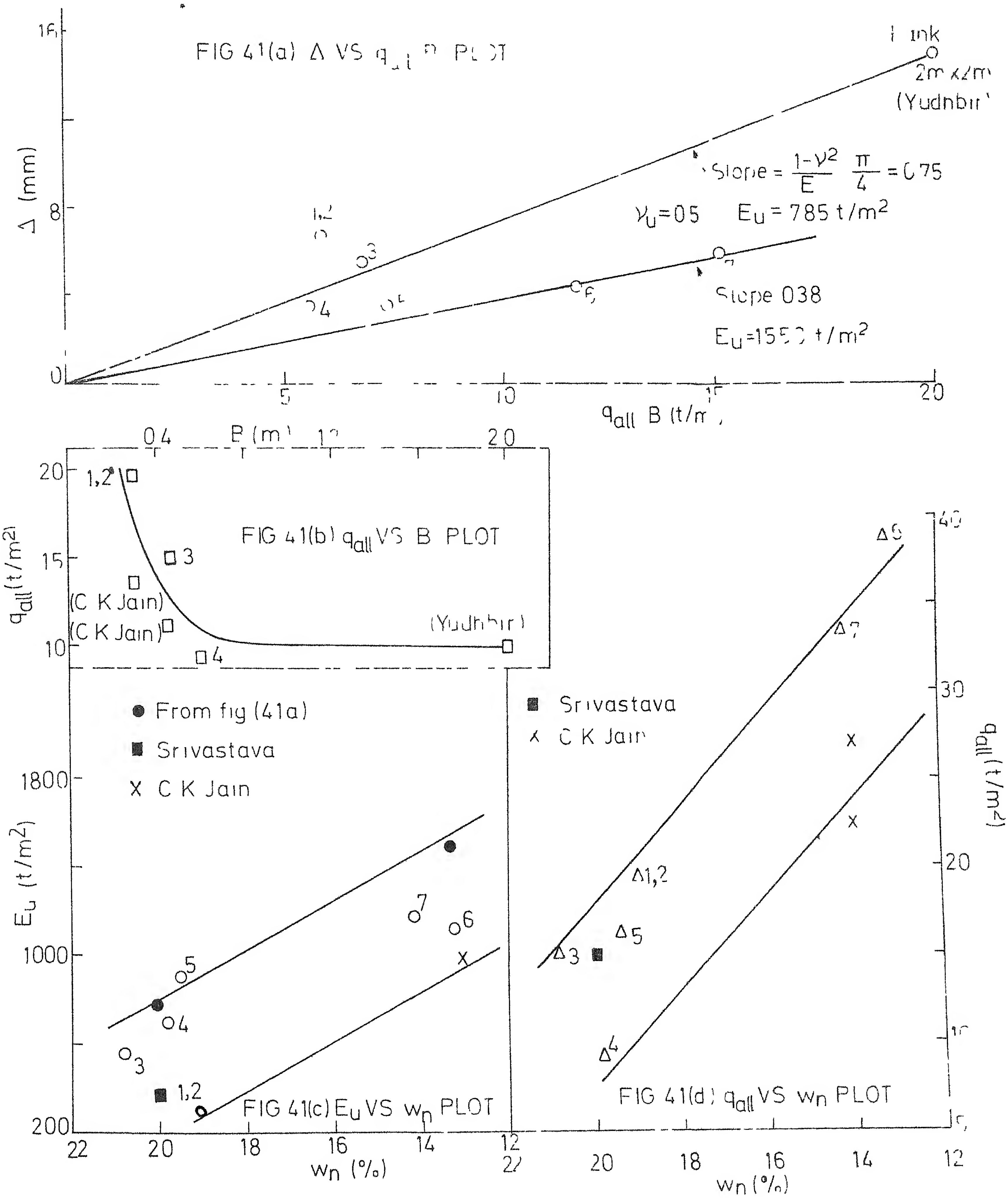


FIG 41(a) Δ VS q PLOT
Δ (mm)
(Yudhbir)
Slope = (1-ν²)/E π/4 = 0.75
ν_u = 0.5
E_u = 785 t/m²
Slope 0.38
q_all B (t/m)
FIG 41(b) q_all VS B PLOT
q_all (t/m²)
B (m)
(C K Jain)
(C K Jain)
From fig (41a)
Srivastava
C K Jain
E_u (t/m²)
FIG 41(c) E_u VS w_n PLOT
w_n (%)
Srivastava
C K Jain
q_all (t/m²)
FIG 41(d) q_all VS w_n PLOT
w_n (%)

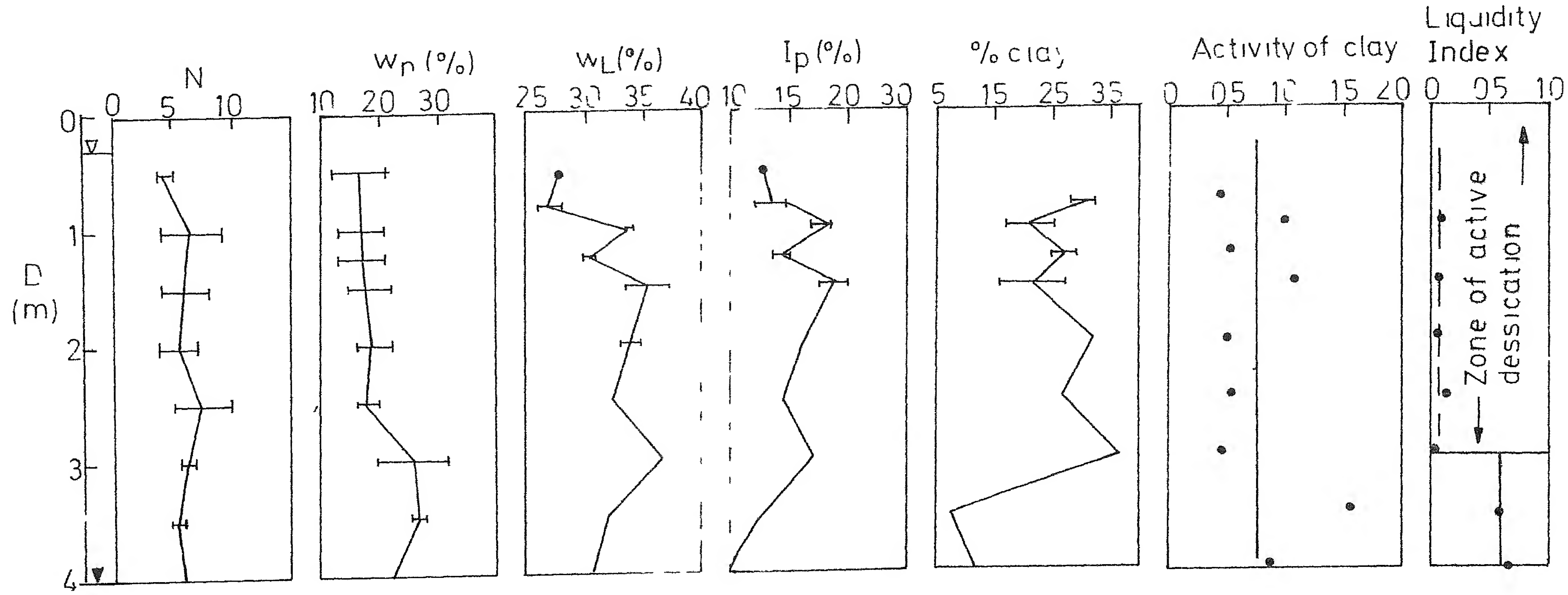


FIG 51 A REPRESENTATIVE SOIL PROFILE FOR SHALLOW FOUNDATIONS IN SOUTH-WESTERN PART OF IIT KANPUR CAMPUS

A 54908

Date Slip A 54908

This book is to be returned on the date last stamped

CD 6729

CE-1978-M-JAI-EVA